Appendix F
Effect of Vertical Shear (Drag) on the Stability of Gravity Walls

F-1. Purpose

Appendix F provides guidance for calculating a shear force acting along the backs of gravity earth retaining walls founded on rock. This shear force is referred to as a downdrag force or simply a drag force. Drag force is not the same as wall friction (Chapter 5) nor is it to be used in conjunction with wall friction. The reasons for not including drag force with wall friction are that the drag force is not related to the lateral earth force but instead is due to the relative settlement of the backfill with respect to the wall, and it is not due to soil compaction during construction. In fact, it is shown later in this appendix that the vertical shear (drag force) that develops is greater for loose backfill than it is for dense backfill. The drag force is an optional external force that may be applied to the back of the structural wedge when determining the stability of a wall on rock using either the multiple-wedge method or single-wedge method of analysis. The following guidance illustrates how to calculate the magnitude of the drag force and how to apply the force to the back of the wall.

F-2. Background

a. Calculation of the stability of gravity walls. A common procedure used for designing new gravity walls and for evaluating the safety of existing gravity walls is the conventional equilibrium method of analysis. The conventional equilibrium method involves assumptions regarding the loading and resisting forces that act on the structure. In most cases of massive retaining walls constructed on rock foundations, movements of the wall and backfill are not sufficient to fully mobilize the shear resistance of the soil. Past practice has been to assign at-rest lateral earth pressures against the back of the gravity wall and set the interface friction between the wall and the backfill equal to zero. Zero interface friction along the back of the wall corresponds to a zero shear force along the back of the wall. In addition, boundary water pressures were assigned along the back, front, and base of the wall for navigation structures. With all forces and their points of action on the free-body diagram of the wall defined, wall stability was checked against the recommended criteria.

b. Shear force-finite element analysis. To develop an improved understanding of the interaction between gravity walls, their foundations, and their backfills, an investigation using finite-element analyses was conducted (Ebeling et al. 1992; Ebeling, Duncan, and Clough 1990). The analyses demonstrated that the backfill settles relative to the wall and develops downward-shear loads on the wall. Some examples are given in Figure F-1, which shows the results of finite-element analyses of four walls founded on rock and retaining dry backfill. In Figure F-1, the magnitude of the vertical shear force, F_V, is expressed in terms of a vertical shear coefficient, K_V, which is related to the shear force on the vertical plane through the heel of a wall by the following equation:

$$F_V = K_V \left(\frac{\gamma_t \, H^2}{2} \right) \tag{F-1}$$

where

γ_t = total unit weight of backfill

H = wall height

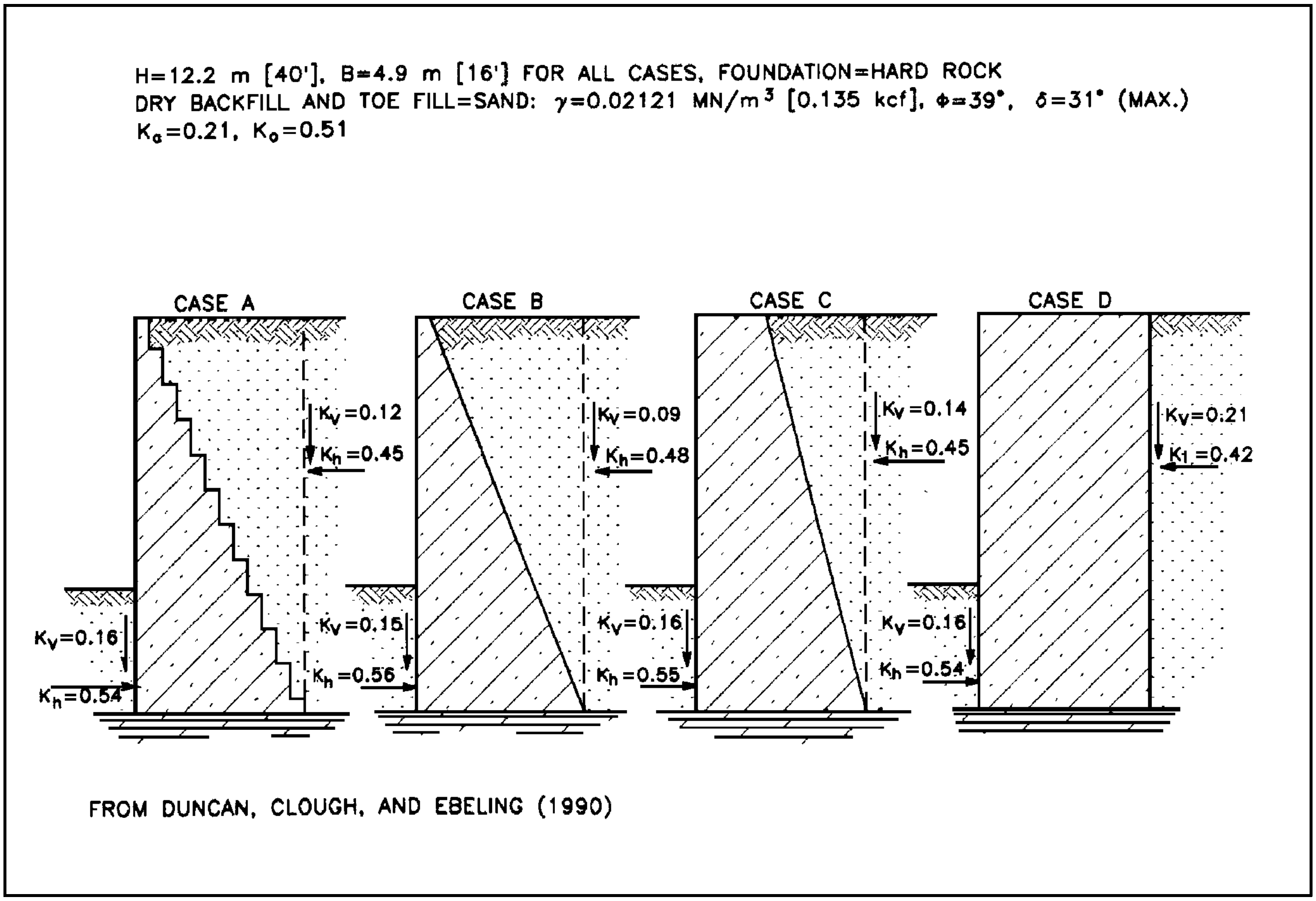


Figure F-1. Results of finite element analyses of four walls founded on rock-retaining dry backfill

Analyses indicated that the gravity walls would move only slightly during the placement of the toe fills and backfills. As a result, the earth pressures on the backs and fronts of the walls are close to those that exist at rest. Even so, settlement of the backfill relative to the wall as it is placed behind the wall is sufficient to generate a significant amount of shear force on the wall. Values of K_V range from 0.09 to 0.21 for the four cases shown in Figure F-1.

Parametric studies demonstrated that the most important factors influencing the value of K_V for concrete gravity walls on rock foundations are the depth of the backfill, the stiffness of the backfill, the inclination of the back of the wall, and the number of steps in the back of the wall. The following trends were observed:

(1) For low walls, the value of K_V increases with increasing wall height because more backfill compression occurs due to self-weight of the backfill. The resulting increase in differential movement between backfill and the wall causes a greater portion of the interface strength to be mobilized. This process approaches a limiting condition for high walls as the interface strength becomes fully mobilized over most of the wall-backfill contact area.

(2) As the stiffness of the backfill increases, backfill compression decreases, and the wall height necessary to mobilize the full interface strength increases. For low walls with vertical back sides, the value of K_V decreases as the backfill stiffness increases.

(3) The value of K_V decreases as the back side of the wall becomes inclined away from the backfill and towards the front of the wall.

(4) The value of K_V is greater for a wall with a stepped-back side than for a wall with a smooth-back side at the same average slope.

c. Shear force-instrumented field and model wall measurements. Shear loads have been reported for several instrumented walls (Duncan, Clough, and Ebeling 1990), including a lock wall 30.2 m (99.1 ft) in height and founded on rock (Hilmer 1986). Measurements at the lock wall are reported over a 6-year period. Mobilized interface friction at the lock wall fluctuates seasonally and with changes in the water level inside the lock. However, the data indicate that the shear force is persistent over the 6-year period and does not decay with time. According to a conservative interpretation of the data, the minimum value of K_V during the 6-year period is about 0.18.

In a recent research program conducted at Virginia Polytechnic Institute and State University (Filz and Duncan 1992), both the horizontal earth pressure force and the vertical shear force along the vertical back side of a 2.1 m (7 ft) high rigid retaining wall were measured. The research program included 16 tests using compacted fine sand (Unified Soil Classification SP) and compacted non-plastic silty sand (SM) as backfill. Measured values of K_V ranged from 0.11 to 0.23 (Table 8.9 in Filz and Duncan 1992). The more compressible backfills exhibited higher K_V values. The compacted backfills were left in place for periods ranging from 1 to 14 days after completion of backfilling. Values of K_V tended to increase with time.

F-3. Procedures for Calculating the Vertical Shear Force

Two procedures for computing the magnitudes of shear loads along the backs of gravity walls are described in this section: a simplified procedure and a complete soil-structure interaction analysis procedure using the finite element method. These procedures are intended only as guidelines and are not intended to replace judgment by the project engineers .

a. Simplified procedure. Inclusion of a shear force on a vertical plane through the heel of the wall, as shown in Figure F-2, can be computed using the following equations:

$$F_V = K_V \left[\frac{1}{2} \gamma_t D_1^2 + \gamma_t (D_1 D_2) + \frac{1}{2} \gamma_b D_2^2 \right] \qquad \text{(F-2)}$$

where

D_1 = thickness of backfill above water table

D_2 = thickness of submerged backfill above the base of the wall

γ_b = buoyant unit weight of submerged backfill = $\gamma_t - \gamma_w$

γ_w = unit weight of water

As indicated in Figure F-2, the total height of the backfill against the wall is the sum of the thicknesses D_1 and D_2:

$H = D_1 + D_2$

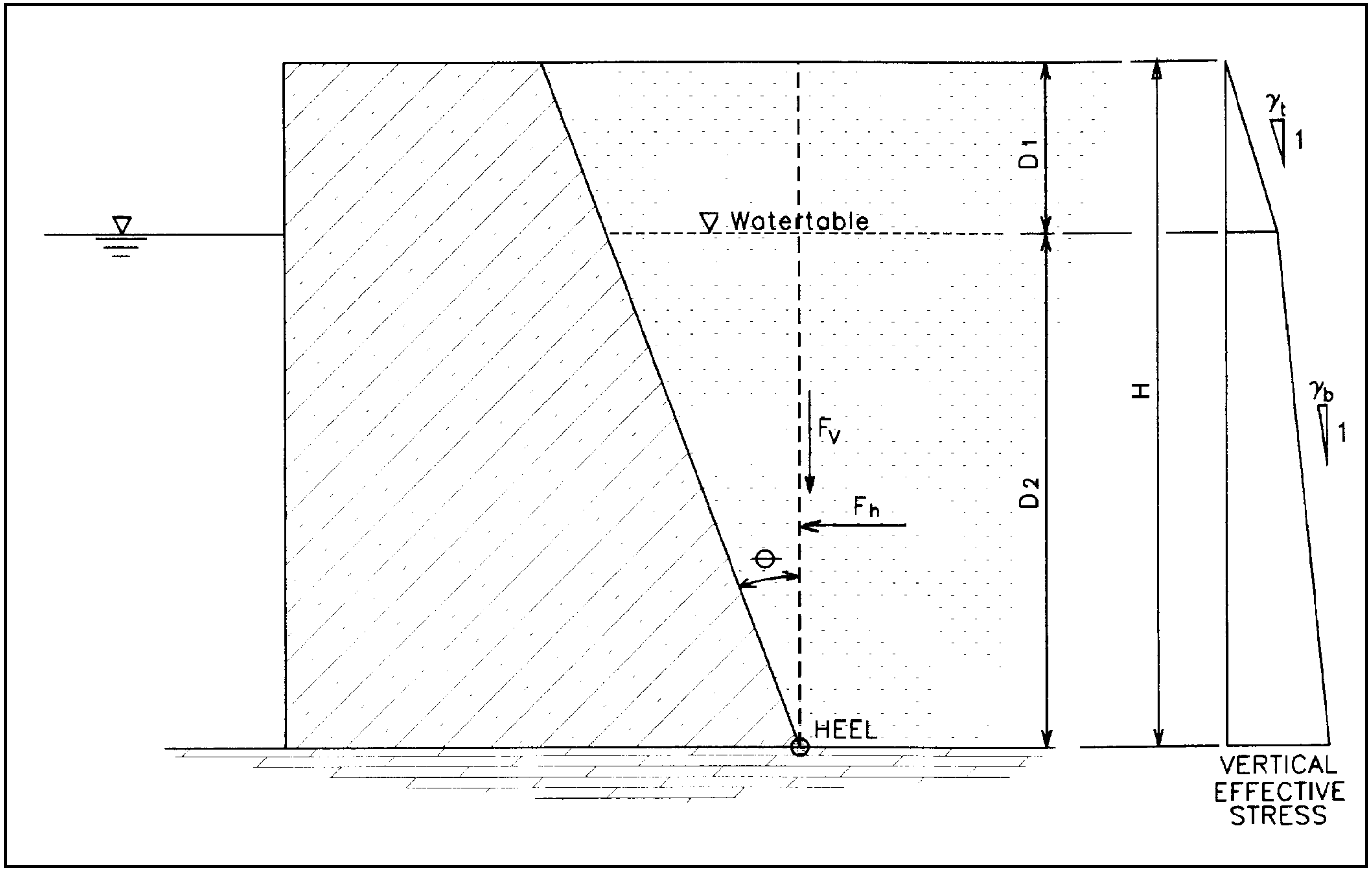


Figure F-2. Resultant earth pressure forces on a vertical plane through the heel of a gravity wall founded on rock

Equation F-2 requires a value for K_V. In the simplified procedure, the value of K_V is obtained from Figures F-3 through F-5 and Equation F-3:

$$K_V = (1 - C_\theta\ C_s\ )\ (K_V)_{vertical} \tag{F-3}$$

where

C_θ = correction factor for inclination of the back side of the wall

C_s = correction factor for steps on the back side of wall

(K_V)vertical = value of K_V for a wall with a vertical back

Figure F-3 shows that the value of $(K_V)_{\mathrm{vertical}}$ for design increases with increasing wall height until a limiting value of 0.1 is reached. This limiting value for design is well below the actual limiting value of $(K_V)_{\mathrm{vertical}}$ indicated by measurements and analyses. It was selected conservatively so that the change to previous design procedures (i.e., $K_V = 0$) would not be large. Even with this conservative selection of the limiting value, significant economies can be obtained by including the vertical shear force in design.

Figure F-3 also shows that the limiting value of $(K_V)_{\mathrm{vertical}}$ develops at lower heights for walls with loose backfill than for walls with dense backfill.

Figures F-4 and F-5 show the values of the correction factors C_Θ and C_s, respectively, that are to be applied.

An example application of the simplified procedure is shown in Figure F-6. It can be noted that the steps in the back side of the wall in the figure are not uniform. An average slope consistent with the definition sketch in Figure F-5 is used to obtain the value of correction factor C_Θ from Figure F-4.

The vertical shear force determined using the simplified procedure can be incorporated in conventional equilibrium calculations. The results should be checked against the required criteria. When a toe fill of significant height exists, a vertical shear force at the toe should be included in the equilibrium calculations if a vertical shear force was applied to the back side of the wall. Neglecting the shear force at the toe could result in unconservative estimates of the base contact area and the maximum bearing pressure on the foundation.

Use of the simplified procedure to obtain a vertical shear force for stability calculations is restricted to gravity earth retaining walls that satisfy the following criteria:

(1) The vertical displacements within the foundation during construction of the wall and backfilling are negligible when compared with the vertical settlement within the backfill due to self-weight. Gravity walls founded on competent rock foundations satisfy this criterion.

(2) The backfill does not creep. Compacted soils classified as SW, SP, GW, and GP according to the Unified Soil Classification System (ASTM 1990) do not experience significant creep movements. The simplified procedure is also applicable to select SM backfills with nonplastic fines that do not creep.

(3) No special features that reduce or eliminate interface friction exist along the interface between the back of the wall and the backfill. Examples of special features that would reduce interface friction include bituminous coatings and synthetic barriers with low interface friction values.

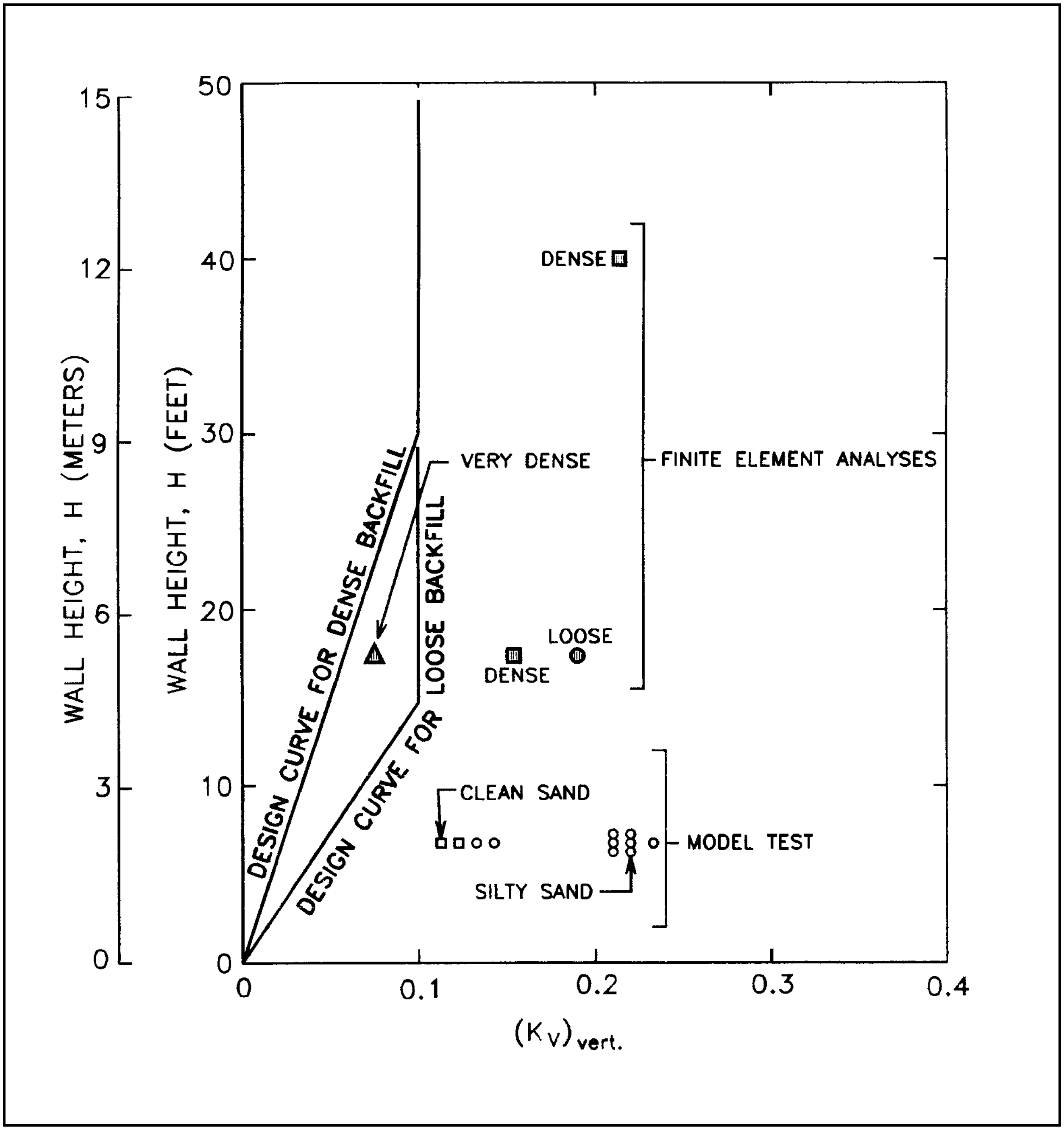


Figure F-3. Values of $(K_V)_{vert}$ for design of gravity walls founded on rock

(4) The interface between the back side of the wall and the backfill is capable of developing friction values of $\delta > 0.7\ \phi$, where ϕ is the effective angle of internal friction for the backfill. This is satisfied by SW, SP, GW, and GP soils compacted against concrete walls. It is also satisfied by SM soils with nonplastic fines compacted against concrete walls.

(5) The water table within the backfill is hydrostatic. If the variation of pore water pressure is not hydrostatic, the values of D_1 and D_2 should be selected to represent the average conditions in the backfill.

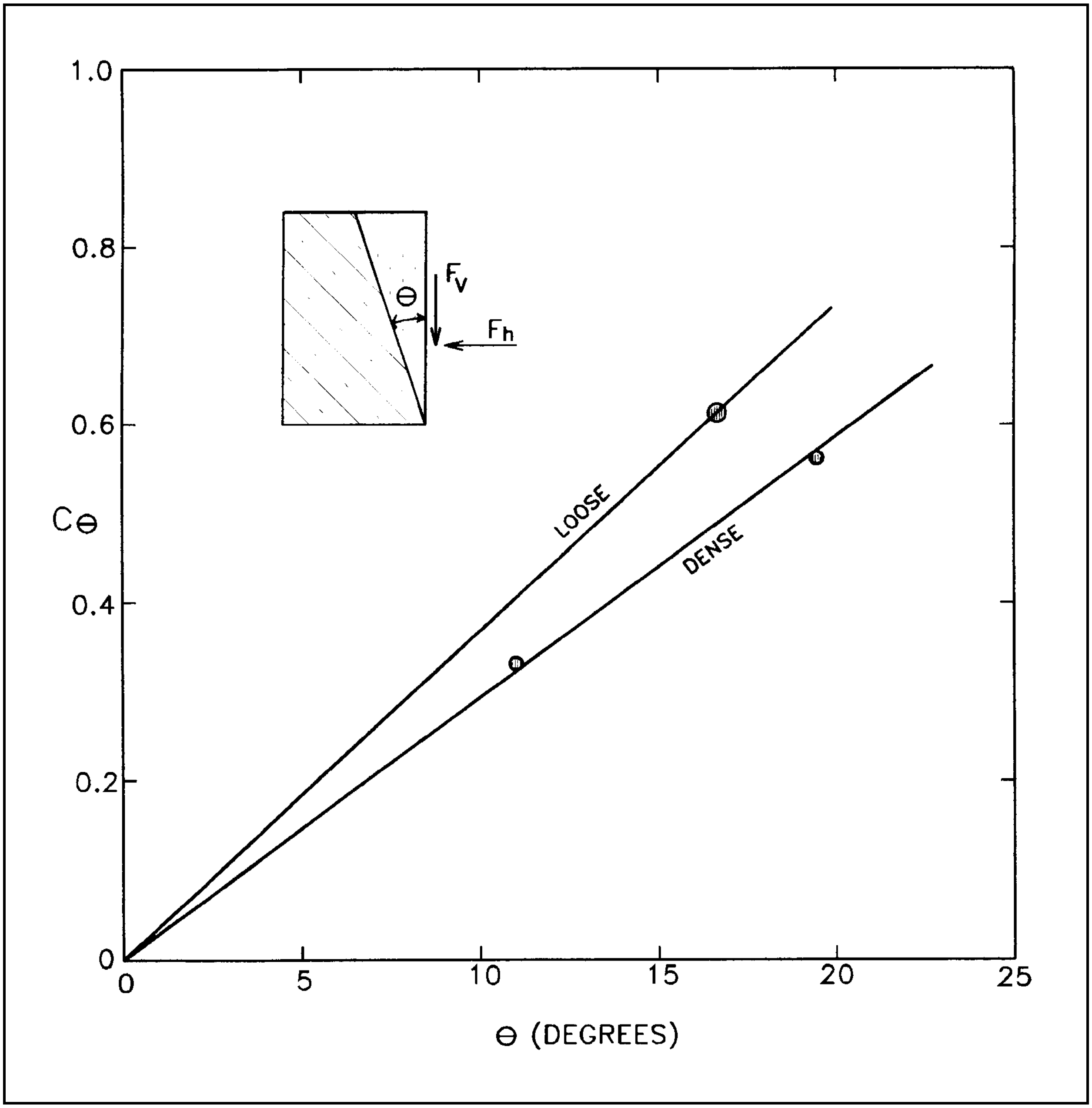
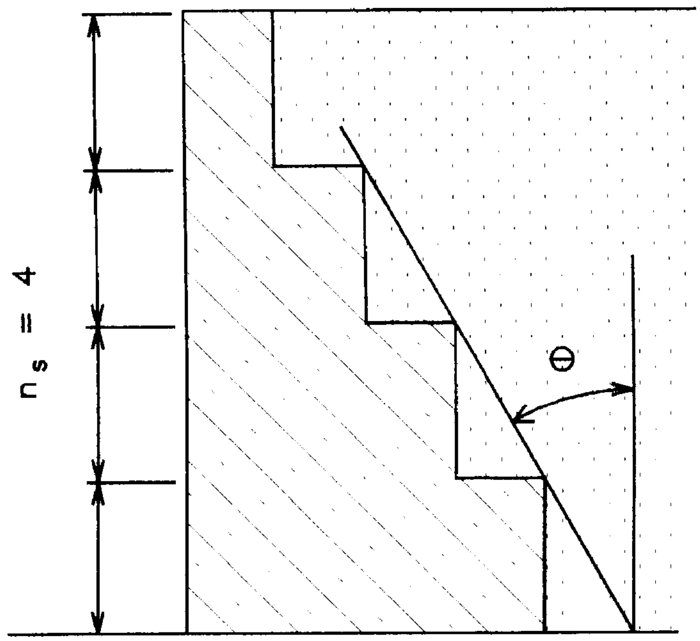


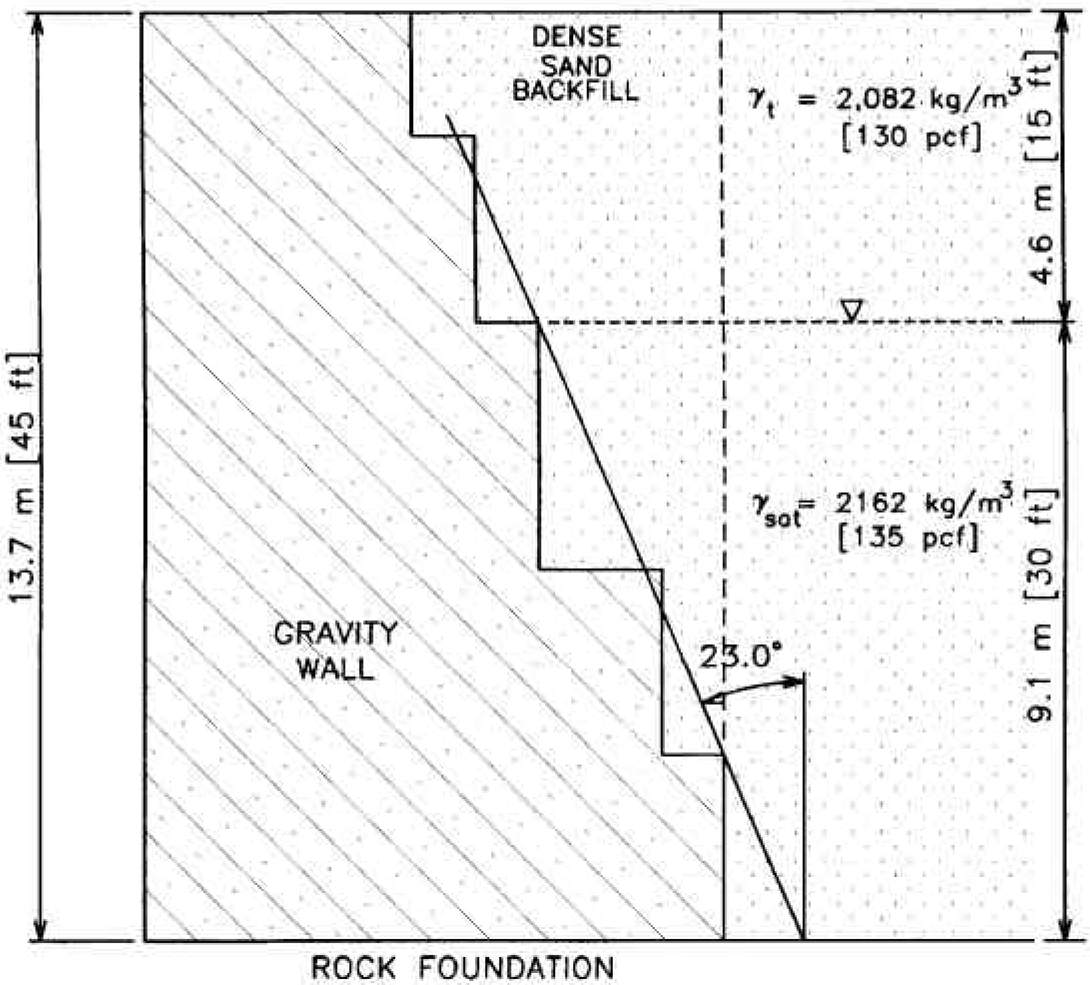
Figure F-4. Values of the correction factor C_1 for inclination of the back side of gravity walls founded on rock

b. Soil-structure interaction analysis. A complete soil-structure interaction (SSI) analysis (Ebeling 1990) for computing shear loads along the backs of gravity walls can be accomplished using a finite element program such as SOILSTRUCT (Ebeling, Peters, and Clough 1992). Unlike conventional equilibrium procedures, an SSI analysis does not require the use of predetermined pressure distributions between the soil and the wall. Instead, it allows for development of these pressures through soil-structure interaction by simulating the staged construction that occurs. The computer program SOILSTRUCT can model the nonlinear stress-strain behavior of the soil and allow for relative movement between the soil and the structure by incorporating interface elements in the mesh.



n_s	C_s
2	0.16
3	0.28
4	0.38
5	0.46
8	0.64
10	0.71
16	0.83
32	0.95
Planar	1.00

Figure F-5. Definition sketch and values of the correction factor C_s for gravity walls founded on rock with stepped-back sides



$H = 13.7$ m [45 ft], $(K_V)_{vert} = 0.1$ [FIG. F-3]

$\Theta = 0.4$ RAD (23°), $C_\Theta = 0.67$ [FIG. F-4]

$n_s = 5$, $C_s = 0.46$ [FIG. F-5]

$$K_V = (1 - C_\Theta C_s)(K_V)_{vert}$$
$$= [1 - (0.67)(0.46)](0.1)$$
$$= 0.07$$

$$F_V = K_V\,(1/2\,\gamma_t\,D_1^2 + 1/2\,\gamma_t\,D_1\,D_2 + 1/2\,\gamma_D\,D_2^2)$$
$$= 0.07[(1/2)(130)(15)^2 + 130(15)(30) + (1/2)(135 - 62.4)(30)^2]$$
$$= 106{,}535 \text{ N/m} \quad [7{,}300 \text{ lbs/ft}]$$

Figure F-6. Example calculation of the vertical shear force for a gravity wall founded on rock

SSI analyses are also especially useful for analyzing retaining structures founded on either soils or compressible rock foundations. Differential settlements within the foundation affect the magnitude of the shear force that the backfill exerts on the wall. The SSI analysis procedure has been successfully used for a wide variety of problems, including the Port Allen and Old River locks (Clough and Duncan 1969) and, more recently, the lock at Red River Lock and Dam No. 1 (Ebeling et al. 1993).

A SSI analysis is recommended for those structures for which the simplified procedure is not applicable, or for those cases in which a more precise evaluation of the shear force is required. Soil-structure analyses are recommended for U-frame locks, retaining structures founded on soils, and structures with complicated geometry.